NON-PRESTRESSED PANEL-TO-PANEL CONNECTION DETAILS FOR FULL-DEPTH PRECAST DECK PANELS

Sameh S. Badie, PhD, PE, The George Washington University, Washington DC, USA
Amgad F. Girgis, PhD, University of Nebraska-Lincoln, Nebraska, USA
Krissachai Sriboonma, DSc, The George Washington University, Washington DC, USA
Maher K. Tadros, PhD, PE, University of Nebraska-Lincoln, Nebraska, USA

ABSTRACT

Transverse panel-to-panel connection details for full-depth precast concrete bridge deck panel systems are considered one of the major challenges facing design engineers. The longitudinal reinforcement of the deck of a slab/I-girder bridge is typically used for providing for the required shrinkage and temperature reinforcement, distributing the concentrated live load, and provide for the negative moment reinforcement required for continuous span bridges. The longitudinal reinforcement of the deck has to be spliced at every panel-to-panel joint and be adequately anchored to develop its yield strength. Many techniques have been developed to accomplish this task.

In the NCHRP 12-65 project completed in early 2007, the research team developed new non-prestressed panel-to-panel connection details for full-depth precast deck panels. The details satisfy the following conditions: minimum development length using a unique confining technique, minimum exposed surface area of the grout filling the joint, minimum size gap between panels (2 in.), and minimum volume of cast-in-place grout/concrete used to fill the joint.

The paper presents a summary of the details and the experimental investigation that was used to confirm their structural feasibility. The experimental investigation included direct tension and fatigue testing. Also, the paper gives a mathematical model that can be used to estimate the development length of confined reinforcing bars.

Keywords: Precast, Deck panels, Bridges, Non-prestressed, Panel-to-panel connection

INTRODUCTION

The use of full-depth precast concrete deck panels in highway bridges in the United States started as early as 1965[1]. The motive behind using this construction system has been to increase the speed of construction of the deck for rehabilitation projects, especially in areas with high traffic volume where traffic closures have high costs and cause inconvenience to the public. Over the years, design engineers have started to see that this construction system is advantageous not only for rehabilitation projects but also for new construction. This is due to the relatively high construction speed and higher quality of precast decks that minimize future maintenance costs and increase their service life.

Typically, full-depth precast concrete deck panel systems supported on I-girders contain transverse (normal to traffic) and longitudinal (parallel to traffic) reinforcement[2]. Transverse reinforcement is used to resist flexural effects due to applied loads. Longitudinal reinforcement in deck slabs is used to distribute the concentrated live load in the longitudinal direction and to provide for required reinforcement to resist shrinkage and temperature effects. Also, longitudinal reinforcement is used to resist the negative bending moment due to superimposed dead and live loads at the intermediate supports of continuous span bridges.

For deck slabs made with full depth precast panels, splicing this reinforcement at the transverse joint between panels is a challenge for design engineers because:

(1) The panels are relatively short, 8 to 10 ft long. Therefore, a wide cast-in-place (CIP) concrete closure joint (2 to 3 ft wide) is needed if the longitudinal reinforcement splices were to be lapped. Wide CIP panel-to-panel joints would defeat the main goal of having a precast deck panel system. This would require wood forming under the panels and extended period of time for curing.

(2) Since the longitudinal reinforcement is spliced at every panel-to-panel transverse grouted-joint, great care has to be taken in detailing the splice connection to protect it from leakage, which can cause the reinforcement to corrode. At the same time, the connection detail has to provide acceptable level of construction tolerance.

(3) Splicing the longitudinal reinforcement requires a high level of quality control during fabrication of the precast panels to guarantee that the spliced bars will match within a very small tolerance.

(4) Splicing the longitudinal reinforcement requires creating pockets and/or modifying the side form of the panels, which increase the fabrication cost.

Various methods have been used to splice the longitudinal reinforcement[3]. These are:

(1) Using a lap splice: Fig. 1 shows an example of this detail where it was used on the precast deck panel system of the C-437 of the County Road Bridge over I-80 to Wanship, Utah. A 30-in wide CIP closure joint was required to splice the #5 longitudinal bars.

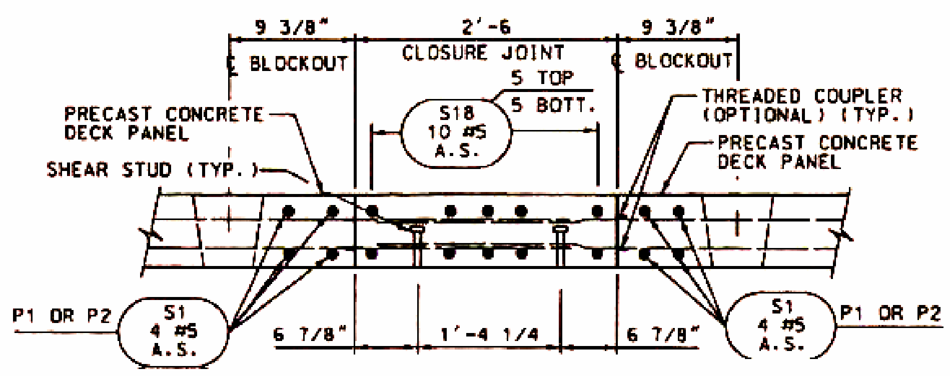


Fig. 1. Lap splicing of longitudinal reinforcement used on Structure C-437, Wanship, Utah

(2) Using U-shaped pin bars: This detail was used on the Castlewood Canyon Bridge in Colorado. Fig. 2 shows the concept where U-shape pins bars are overlapped, confined with rectangular stirrups, and 2 #6 bars were fed into the loop to provide for mechanical interlocking. This detail requires staggering the spliced bars and the spliced bars have to extend outside the side forms of the panel during fabrication. Also, this detail requires feeding the interlocking bars from one side of the bridge, which may not be convenient in some cases.

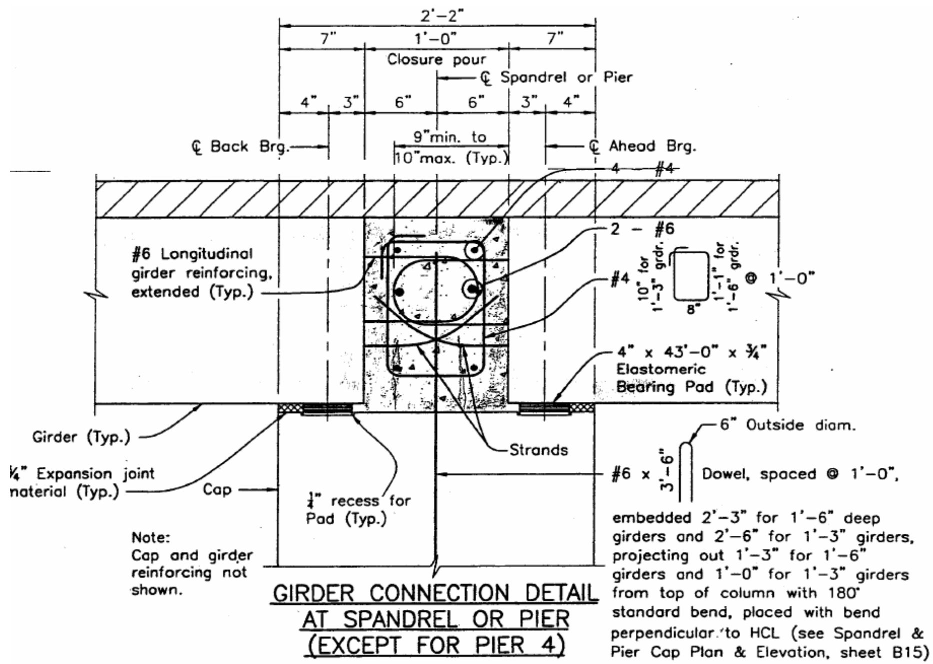


Fig. 2. Continuity detail over the cross piers used on Castlewood Canyon Bridge, Colorado

(3) Using spiral confinement: This detail was developed to reduce the lap splice length of spliced bars and give higher construction flexibility for the spliced connection[4,5]. Fig. 3 shows the spliced connection where a loose bar confined with high strength spiral is used. Experimental investigation of the technique has shown that the lap splice length can be

reduced by about 40 to 50 percent, and simplifies the fabrication of the panel because no bars extend outside the transverse edges of the panel.

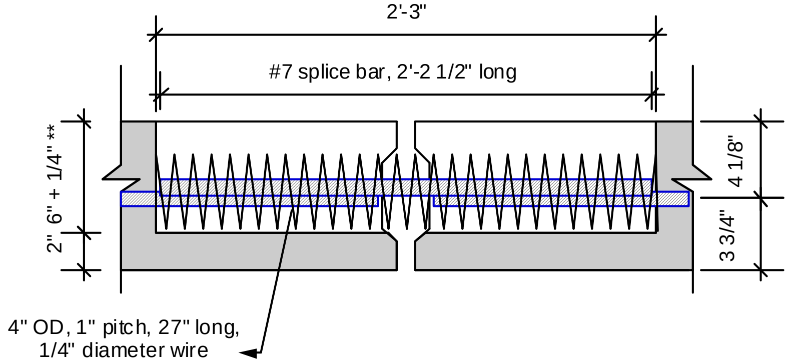


Fig. 3. Panel-to-panel connection using spiral confinement

(4) Using longitudinal post-tensioning: Longitudinal post-tensioning has been used on the majority of bridges built with full-depth precast panels. Longitudinal post-tensioning puts the transverse panel-to-panel joints under compression that eliminates the tensile stresses resulting from live load. The amount of the post-tensioning stress on the concrete after seating losses used in bridge decks ranges from 150 to 250 psi. Longitudinal post-tensioning is typically applied after the transverse panel-to-panel joints are grouted and cured, but before the deck/girder connection is locked. This procedure guarantees that all of the post-tensioning force is applied to the precast deck.

In most cases, high strength threaded rods uniformly distributed between girder-lines are used. The threaded rods are fed through galvanized or polyethylene ducts that are provided in the panels during fabrication. Fig. 4 shows the post-tensioning details that were used on Bridge-4 constructed on Route 75, Sangamon County, Illinois. Longitudinal post-tensioning can be provided in stages and coupled. After the threaded rods are post-tensioned and secured, the ducts are grouted with non-shrink grout to protect the threaded rods from corrosion.

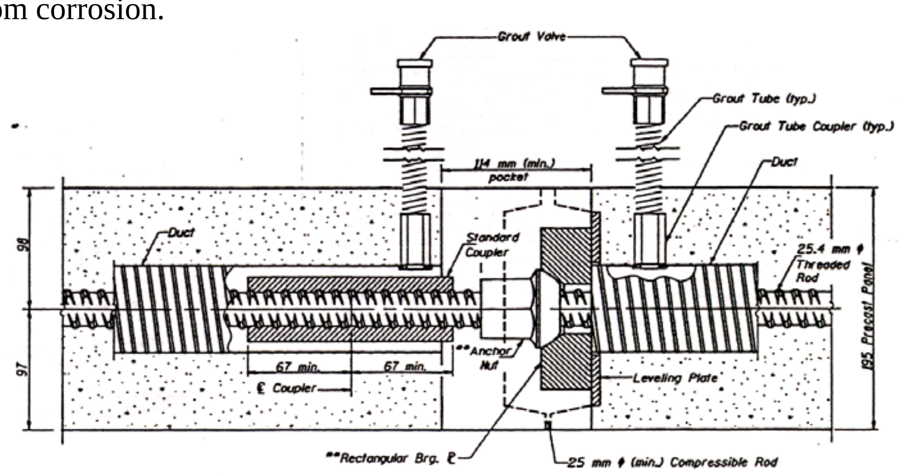


Fig. 4. Post-tensioning detail used on Bridge-4 constructed on Route 75, Sangamon County, Illinois

Recently, longitudinal post-tensioning concentrated at the girder lines has been used on the Skyline Drive Bridge in Omaha, Nebraska[6]. Fig. 5 shows a cross section of the bridge at a girder line. The post-tensioning consists of 16- ½ in. diameter, 270 ksi, low relaxation strands. The strands are fed into open channels created over the girder lines, and a special end panel that houses the anchorage device is used as shown in Fig. 5.

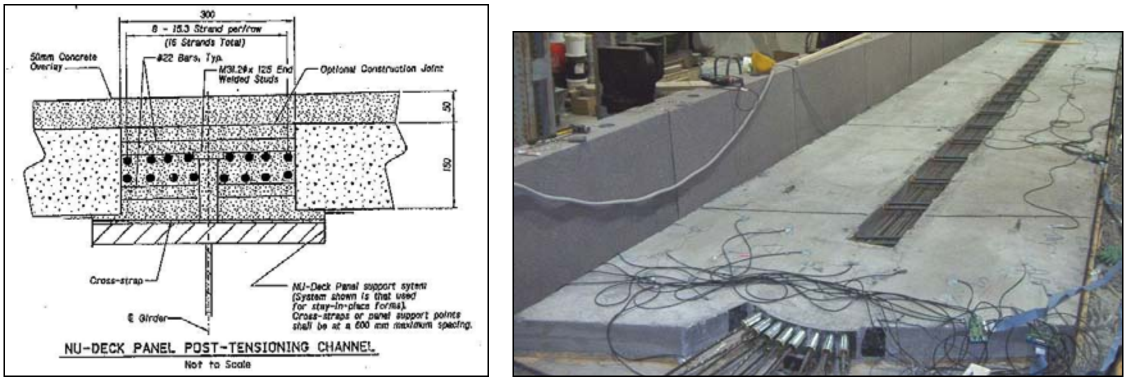


Fig. 5. Longitudinal post-tensioning concentrated at girder lines used on the Skyline Drive Bridge, Omaha, Nebraska

This paper presents two splicing details that were developed in the NCHRP project 12-65[3], titled "Full-Depth, Precast-Concrete Bridge Deck Panel Systems," and sponsored by the National Academies. One of the objectives of the project was to developed new details for panel-to-panel joints. The project criteria mandated that these details should not utilize longitudinal post-tensioning and the precast deck system utilizing these details could be opened to traffic without having any kind of overlays.

DESCRIPTION OF THE NEW DETAILS

DESIGN CRITERIA

Typically, the panel-to-panel connection detail is considered an integral part of any precast deck system. This means that the connection detail has to be developed within a full precast deck system. Therefore, the researchers set the following general criteria in advance to pave the way for the development of a deck system that utilizes the new connection details. These criteria were set after careful study of the bridges covered in the literature review and the national survey of the NCHRP 12-65[3]. Also, these criteria were discussed with a panel of national experts on this type of construction to make sure that they are widely accepted.

1. The slab/I-girder bridge type is used. This decision was made based on the fact that approximately 50 to 60 percent of the bridges in USA are made of this type.
2. The deck slab is made from conventionally or prestressed reinforced concrete. The supporting I-girder can be made of prestressed concrete or steel.
3. The deck system is made composite with the superstructure.

4. The precast deck systems can fit new construction projects as well as deck replacement projects. This decision was made because there is almost a 50/50 percent split between new construction and deck replacement project nationwide.
5. No longitudinal post-tensioning is used: This criterion was mandated by the project problem-statement. Please, note that although these systems were developed without utilizing longitudinal post-tensioning or overlay, they can be easily modified to accept them.
6. No overlay 1s used: This criterion was also mandated by the project problem-statement

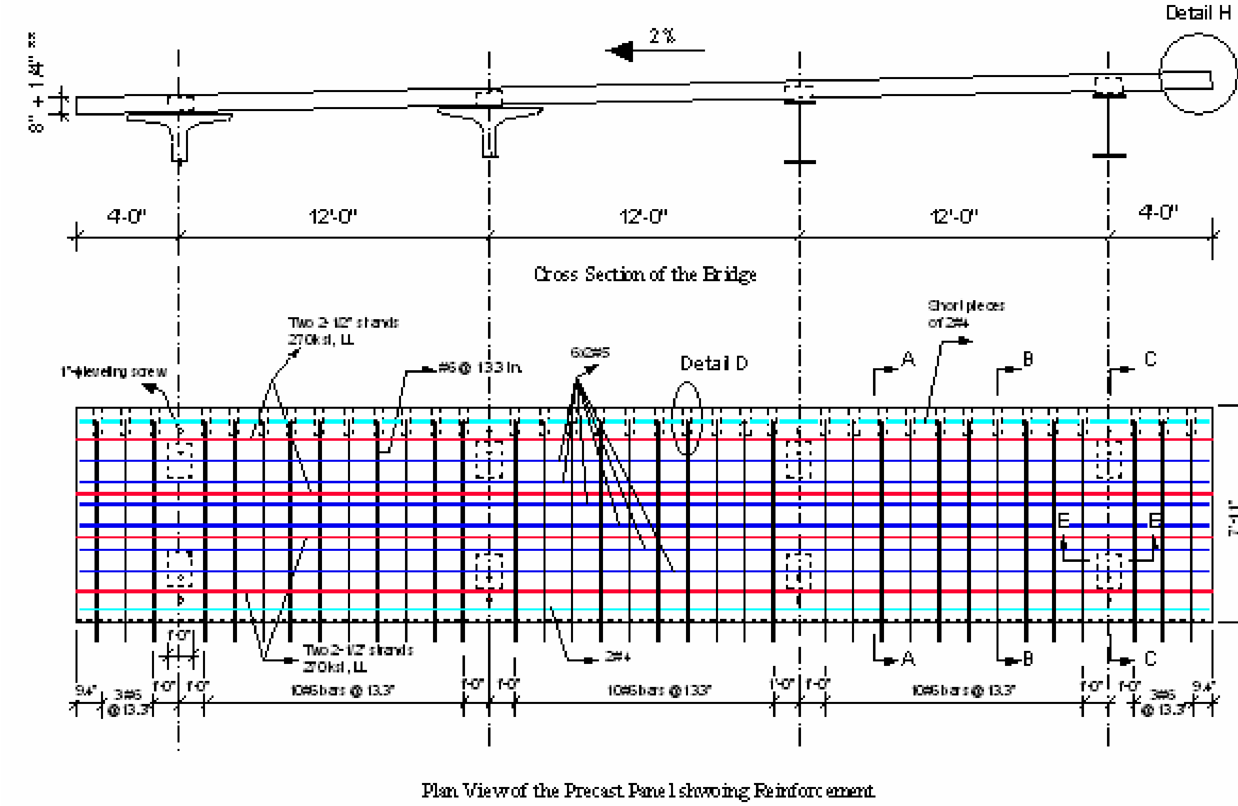
The following model bridge was considered to develop the recommended systems:

Total width	44 ft (13.41 m)
Superstructure	Four steel girders spaced at 12 ft (3.66 m) with top flange width = 12 to 14 in. (300 to 356 mm) *OR* Four BT-72 or NU1800 prestressed precast concrete girders spaced at 12 ft (3.66 m). The 12-ft (3.66 m) girder spacing was chosen to provide extreme straining actions in the deck, and consequently, the highest amount of reinforcement.
Concrete deck panels	Total thickness = 8¼ in. Structural slab thickness = 8 in. Normal weight concrete, unit weight = 150 pcf Compressive strength at 28 days = 6.0 ksi
Grout material	Compressive strength at time of opening the bridge for traffic = 6.0 ksi
Live load	HL-93 (AASHTO LRFD Specifications)
Side barriers	NJ barrier, 600 plf (2.19 kN/m) per side
Design	The deck design is carried in accordance with the AASHTO LRFD Specifications[7].

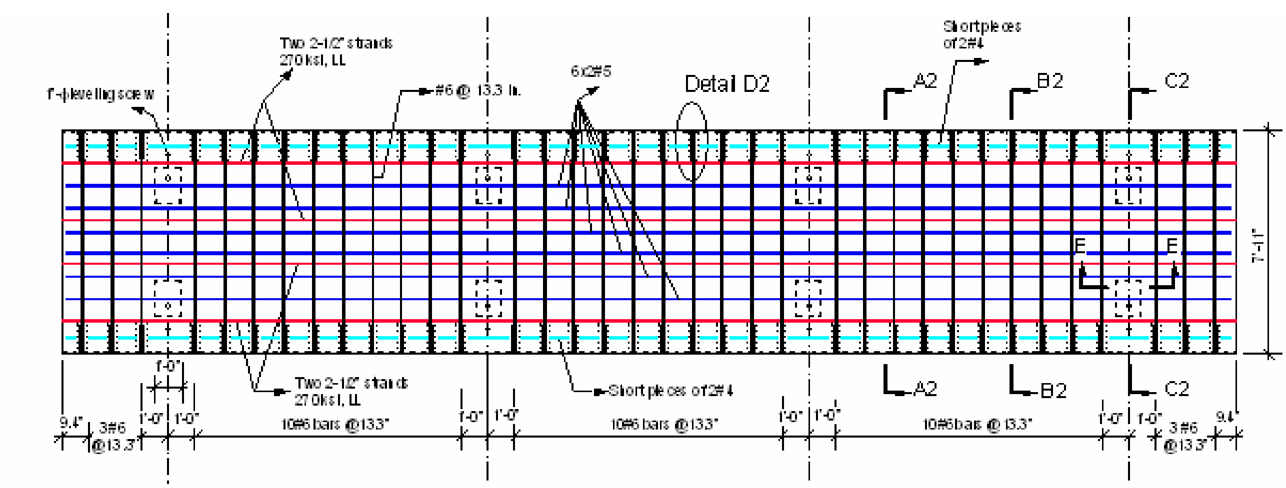
Figs. 6 to 8 show the details of recommended system. The panel is 8-ft long and covers the full width of the bridge, 44 ft. The panel has a design structural thickness of 8 inches. However, the panel is made 8¼ in. thick because no overlay is used. The top ¼ in. of the panel thickness is used as a sacrificial layer for texturing the top surface of slab. Texturing is executed by machine grinding after the panels are installed and grouted. The texturing process helps to have a uniform elevation of the finished deck slab and provides for high quality riding surface.

The panel is transversely reinforced with 8- ½ in. diameter pretensioned strands and 12- No. 5 bars distributed on two levels. A 2-in. top and bottom clear concrete cover is provided for the two layers of reinforcement. This amount of reinforcement is sufficient to cover the required flexural capacities in the positive and negative moment areas.

The longitudinal reinforcement of the panel is made of No. 6 bars at 13.3 in. This amount of reinforcement was determined to satisfy the shrinkage and temperature reinforcement requirement stated by the AASHTO Specifications. In order to splice these bars across the transverse panel-to-panel joints, two connection details were developed, CD-A and CD-B.

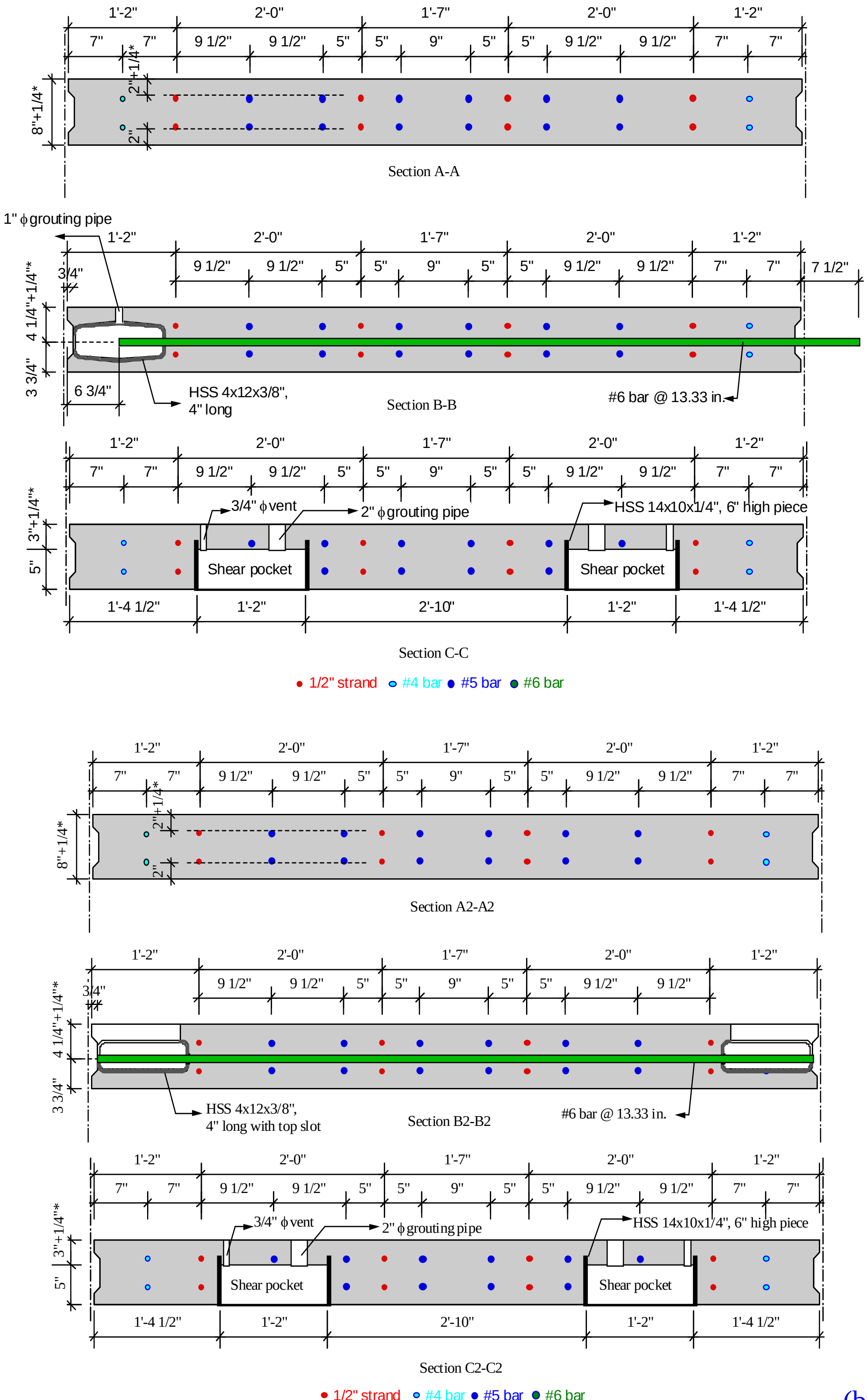


(a) With CD-A



(b) With CD-B

Fig. 6. Cross Section and Plan View of the Precast Deck System
(** 1/4 in. is used as a sacrificial layer for texturing)



(a) With CD-A

(b) With CD-B

Fig. 7. Sections A-A, B-B and C-C

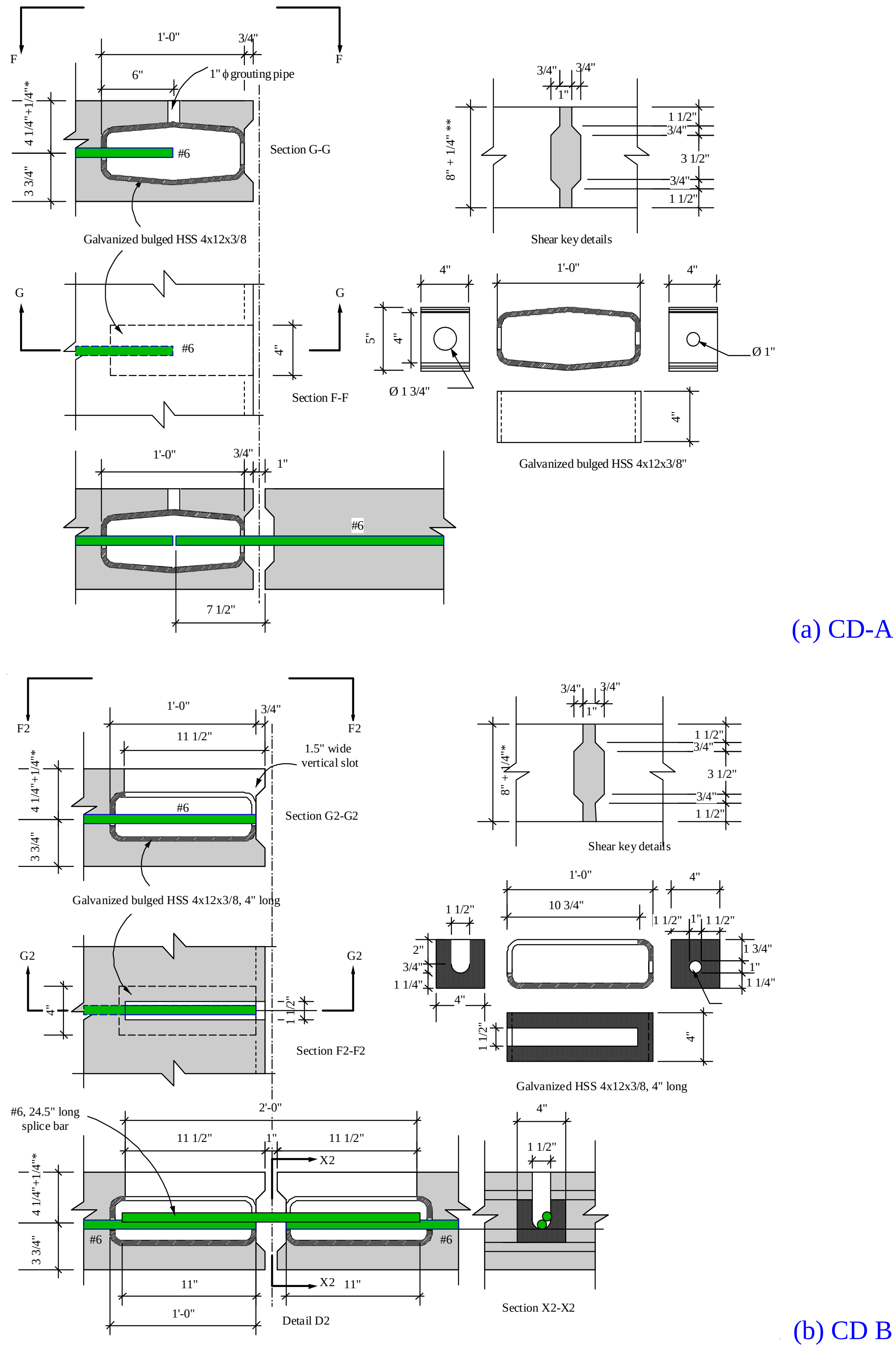


Fig. 8. Details of CD-A & CD-B

Connection Detail A, CD-A (Figs. 6(a), 7(a) & 8)

On one side of the panel, the No. 6 bar is embedded 6 in. in a galvanized bulged hollow structural steel tube, HSS 4x12x3/8 in. On the other side of the panel, the No. 6 bar extends 7½ in. outside the panel. The HSS tube is a 4-in. cut and is installed on its side. It is bulged in the middle to a total height of 5 in. The HSS tube is kept empty during casting of the panel's concrete by covering its sides with thin cardboard sheets. A 1-in. diameter plastic pipe is attached to the top surface of the HSS tube and is used to fill the tube with flowable grout. The HSS tube has an oversize 1¾ in. diameter hole on the free side of the panel to help in installing the new panel without interference with the shear connectors. The panels are installed so that the HSS tubes are ready to receive the No. 6 bars of the next panel. During installation, the new panel, i.e. to be installed, will be tilted in order to avoid interference with the shear connectors of the superstructure.

The tension force is transferred between the spliced bars through bond with the surrounding grout. The steel tube provides confinement to the grout, enhancing its bond with the spliced bars.

Connection Detail B, CD-B (Figs. 6(b), 7(b) & 9)

On both sides of the panel, the No. 6 bar is embedded 12 in. in a HSS 4x12x3/8 in. tube. The dimensions of the HSS tube are exactly the same as those of HSS used in CD-A. However, the HSS tubes are not bulged and they are provided with a 1.5-in. wide top slot. The slot extends all the way to the top surface of the panel. The new panel is installed vertically, and then a 24½ -in. long splice bar is dropped from the top surface of the panel through the slot.

The goal of using the HSS tube is to confine the grout surrounding the No. 6 bar, which enables the bar to develop its yield strength in a distance shorter than that required for unconfined bars. According to the LRFD Specifications[7], an unconfined No. 6 bar requires at least 18 in. to develop its full yield strength. In CD-A, the No. 6 bar has only about 6-in. of embedment length, and in CD-B it has 12 in. of lap splice length.

The new details were tested for direct tension due to static load and for flexure due to repeated (fatigue) loading and the test results have shown full development of the No. 6 bar yield strength, namely 60 ksi.

The panel is made composite with the supporting girder through hidden shear pockets. The shear pockets are 12 in. wide, 14 in. long, 5 in. high, and they are spaced at 48 in. The dimensions of the pockets are optimized to minimize the volume of grout needed to fill the pocket, which eventually will enhance the system economy. The 48-in. spacing of the pockets was chosen to simplify the fabrication process of the panels by minimizing the number of shear pockets to be formed and eventually reducing the fabrication cost. An experimental validation was conducted, through push-off specimens and full-scale beam

testing, because the 48-in. spacing was not in accordance with the LRFD Specifications[7] that limit the spacing to 24 in. Details on this issue can be found in References 3, 8 & 9.
EXPERIMENTAL VALIDATION OF THE NEW CONNECTION DETAILS

Direct Tensile Test (Pullout Specimens)

Twenty five pullout specimens were fabricated and tested for direct static tension. Please note that two additional connection details were developed and tested with the connection details CD-A and CD-B; however, they are not presented in this paper as CD-A and CD-B showed a better performance. The following variables were considered in making the specimens:

- Size of the HSS tube: two sizes were used, which are HSS 3x12x¼ in. (76x305x6 mm) and HSS 4x12x3/8 in. (102x305x10 mm). For both sizes, a 4-in. (102 mm) long strip is used. These sizes were chosen because they fit the 8-in. (203 mm) thickness of the panel, while satisfying the minimum top and bottom concrete cover for reinforcement in deck slabs as specified by the AASHTO Specifications, and they are commercially available from many producers at regular price like the majority of steel products and not considered a special order.

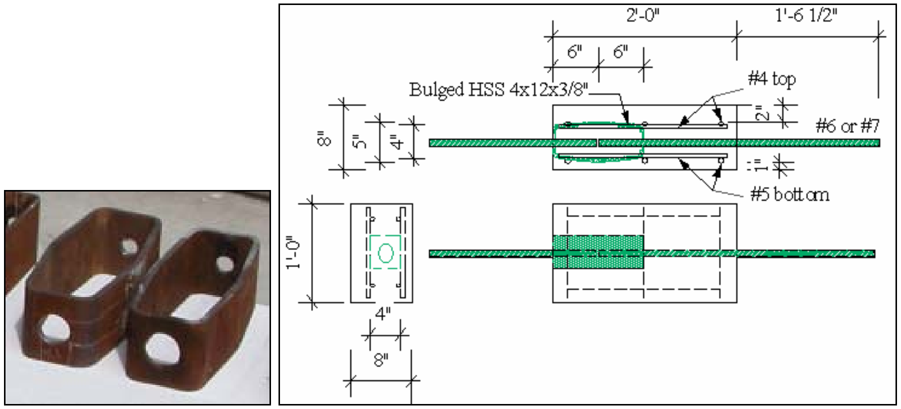
- Size of spliced bar: Two bar sizes were considered, No. 6 (19) and No. 7 (22), Grade 60 (414 MPa) uncoated bars.

Table 1 shows the design criteria of the pullout specimens and Fig. 9 shows the details of the test specimens. Each HSS tube was embedded in 8 in. x12 in. x24 in. concrete prism and the concrete prism was reinforced with two No. 4 top bars and two No. 5 bottom bars. This amount of reinforcement was chosen to simulate the reinforcement required by the empirical design method given by the AASHTO LRFD Specifications[7].

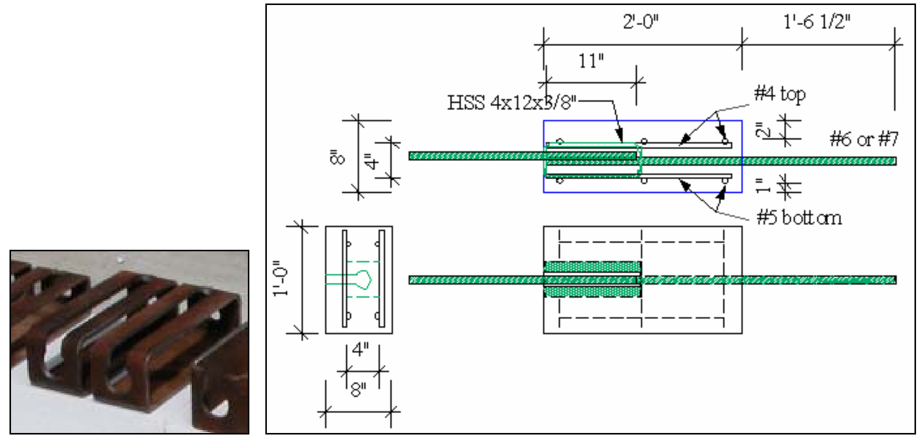
Table 1. Results of the Direct Tensile Test

Detail	**HSS tube (in.)**	**Size of spliced bar**	**Mode of failure**	**Failure load (kips)**	**Developed bar strength, f_d (ksi)**	$\frac{f_d}{60}$ %
CD-A	HSS 4x12x3/8	No. 6	Prism	37.7	85.3	142%
	HSS 4x12x3/8	No. 7	Bar Slip	43.4	72.2	120%
	HSS 3x12x¼	No. 6	Bar Slip	32.8	74.2	124%
	HSS 3x12x¼	No. 7	Bar Slip	45.2	75.2	125%
CD-B	HSS 4x12x3/8	No. 6	Bar Slip	33.8	76.8	128%
	HSS 4x12x3/8	No. 7	Bar Slip	40.9	68.0	113%
	HSS 3x12x¼	No. 6	Bar Slip	32.9	74.8	125%
	HSS 4x12x3/8	No. 7	Bar Slip	40.3	67.0	112%

The tube was set flush with one end of the concrete prism and one of the two spliced bars was embedded in the prism and extended inside the HSS tube to represent the longitudinal reinforcement of the panel.



(a) Specimen CD-A



(b) Specimen CD-B

(c) Specimen CD-B

(d) Test Setup

(e) Slippage Failure

(f) Concrete Failure

Fig. 9. Specimens for the Direct Tensile Test

This bar extended outside the concrete prism from the other end to be hooked with the grip of the testing machine. The HSS tubes were kept empty during concrete casting of the prisms by covering their sides with thin cardboards. A normal weight concrete mix with a specified 28-day concrete strength of 6.0 ksi was used. The specimens were moist cured for 7 days and the measured compressive strength at 28 days was 6.2 ksi.

After 28 days the second bar was embedded in the tube and then the tube was filled with mortar grout. No pea gravel was added to the grout mix. The specimens were tested when the grout was 3-day old using the Tinius Olsen Machine, as shown in Fig. 9(d). The specimens were loaded at a fixed rate of 300 lbs per second until failure. Two modes of failure were observed: (1) Bar slippage, as shown in Fig. 9(e), where the failure load was measured at the moment when the bar started to slip away from the concrete prism, and (2) Concrete failure, as shown in Fig. 9(f), where the concrete around the HSS tube failed in axial tension. Based on the test results that are given in Table 1, the following conclusions were reached:

1. Both details using HSS 4 in. x12 in. x3/8 in. tube and No. 6 bar were able to develop 1.25 times (or greater) the specified minimum yield strength (60 ksi) of the spliced Grade 60 steel bars.
2. Detail CD-A provides higher level of confinement than detail CD-B.
3. No. 7 bars needs longer embedment/splice length to develop the required 1.25 times the specified minimum yield strength (60 ksi)

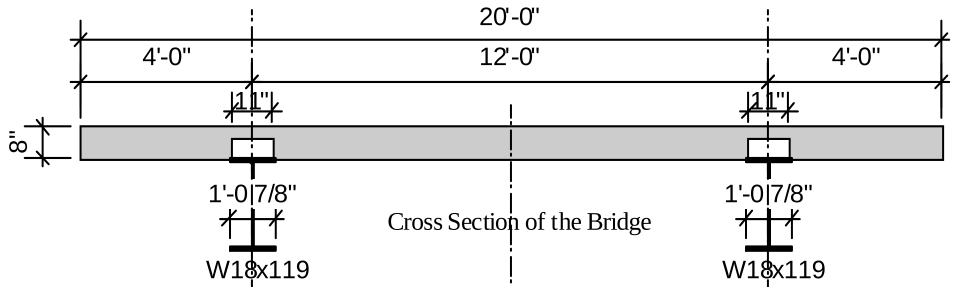
Based on the test results of the pullout specimen, the research team decided to consider the connection details CD-A and CD-B with No. 6 bar and HSS 4 in. x12 in. x3/8 in.

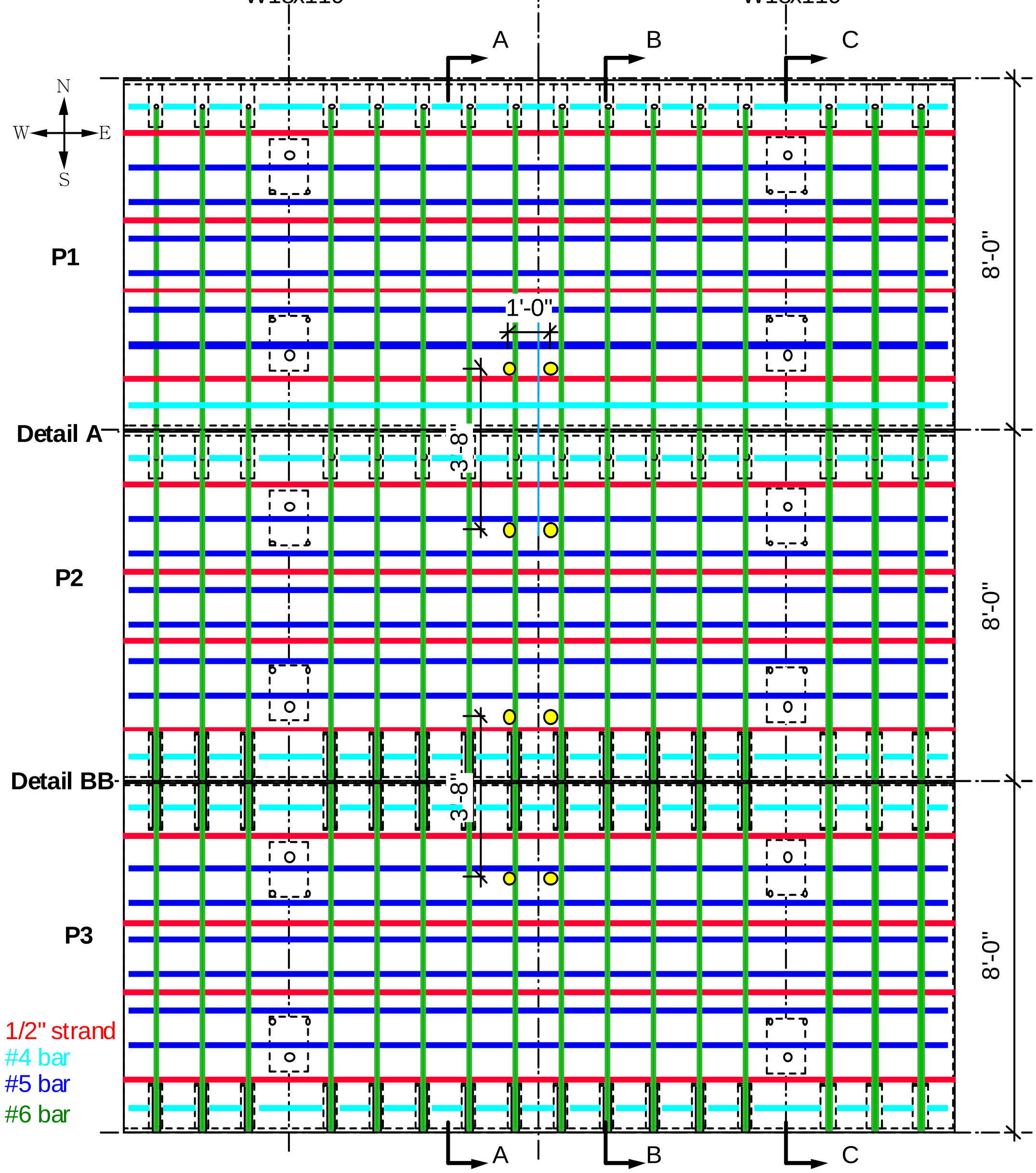
Full-Scale Bridge Test

The bridge was made of 20-ft wide, 24-ft long, 8-in. thick concrete deck supported by two W18x119 steel beams. The steel beams were set at 12 ft on center. The concrete deck was made of three precast concrete panels measuring 20 ft wide x 8 ft long each. The panel-to-panel connection details, CD-A and CD-B, were used on these panels as follows:

Panel P1: CD-A was used on the North and South transverse joints
Panel P2: CD-A was used on the North transverse joint and CD-B was used on the South transverse joint
Panel P3: CD-B was used on the North and South transverse joints

Figs. 10 and 11 show details of the bridge specimen. Fig. 12 shows the precast panels during fabrication and installation. Fig. 12(d) shows the test setup, where a self-equilibrium frame was built at the transverse joint. The self-equilibrium frame consisted of a top and bottom beam connected together with four 2.0 in. diameter high strength threaded rods. A 110-kip hydraulic actuator and a load spreader beam were used to apply the fatigue load. The spreader beam was supported by the precast panel at two points spaced 6 ft (1.82 m) using two neoprene pads, 9x22 in. each.





Plan view showing the reinforcement details

Fig. 10. Cross Section and Plan View of the Full-scale Bridge Specimen

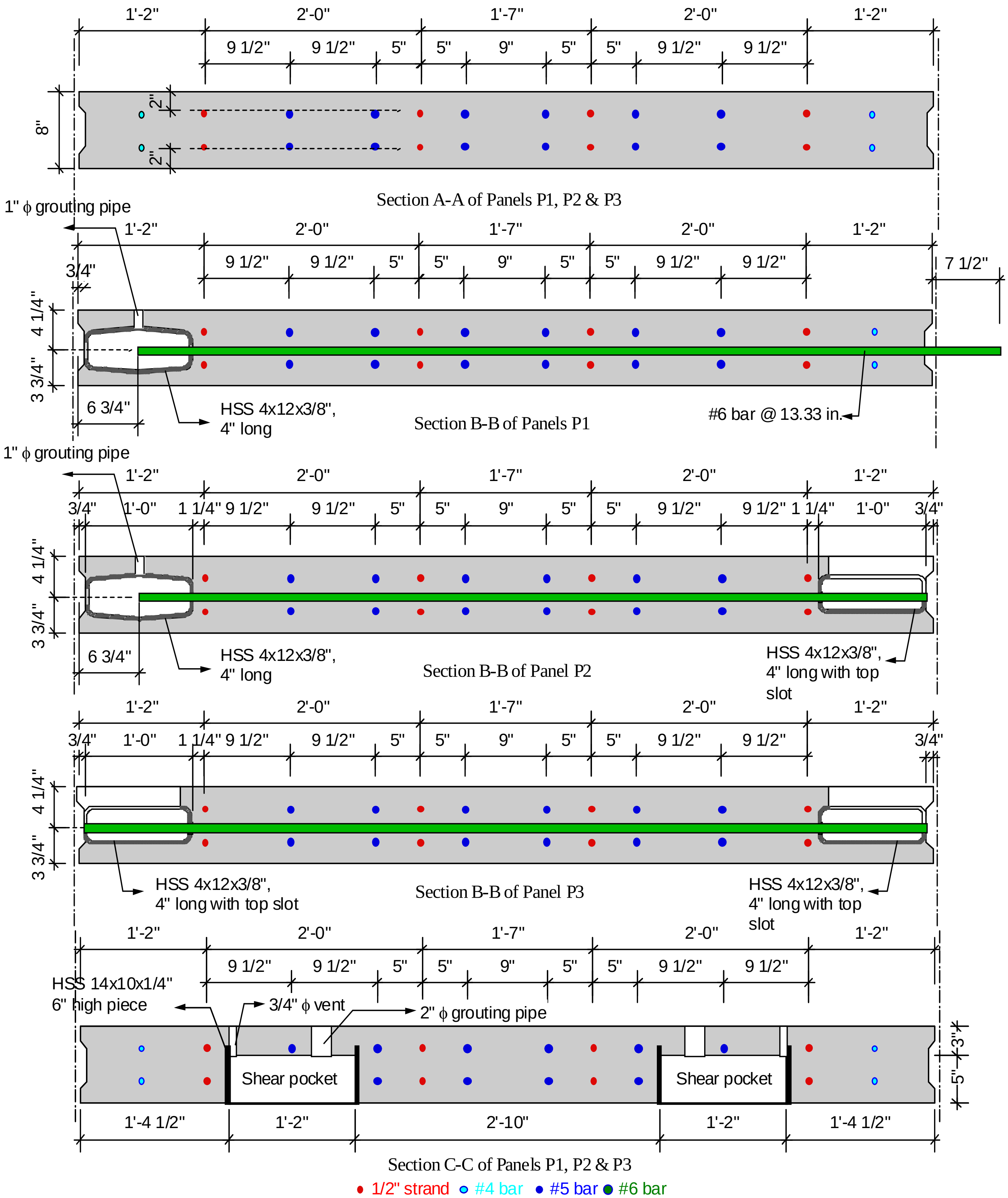


Fig. 11. Sections A-A, B-B & C-C of Panels P1, P2 & P3


(a) CD-A


(b) CD-B

(c) Installation of the Panels


(d) Test Setup

Fig. 12. Fabrication, Installation & Test Setup of the Full-Scale Bridge

The dimensions of the neoprene pads were determined according to the LRFD Specifications[7]. The supports were positioned on one side of the transverse joint. This load arrangement simulated the center axle of HS20 truck. The applied load fluctuated between 4 kips and 46.56 kips (189.3 kN). The 4-kip load was used to maintain stability of the test setup, while the 42.56-kip difference between the high and low load was determined based on the weight of the center axle of HS20 Truck plus dynamic allowance, 32 kips x 1.33 = 42.56 kips. The fatigue load was applied for 2,000,000 cycles at 2 cycles per second as recommended by ASTM D6275-2003[10].

The following steps were taken to build the full-scale bridge specimen: (1) 1-in. diameter backer rods were glued to the top surface of the steel beams to form the haunch between the panels and the steel beams, (2) Panel P2 was installed vertically and set on the steel beams using 1.0-in. thick shims, (3) Panel P1 was lifted from the prestressing bed, and tilted about 15 degrees by shortening the length of the chains on one side of the panel. The panel was successfully installed by inserting the No. 6 bars into the over size holes provided on the transverse side of Panel P2, then the panel was lowered and moved horizontally. The installation process took about two minutes and went smoothly without the need to change the tilting angle of the panel during installation, (4) Panel P3, which had connection CD-B, was installed vertically, (5) Plywood strips were used to form the bottom side of the panel-to-panel joints. The plywood strips were hung from the top surface of the panels using threaded rods, (6) The shear pockets were filled with SS Mortar Grout through the grouting tubes, (7) The transverse shear key joints were filled with the SS Mortar Grout. The grout was flowable enough to set without using any external vibrators.

When the grouting material reached the minimum required strength of 6 ksi, the test setup was built around the north transverse joint P1-P2, between Panel P1 and P2. The load was positioned in the transverse direction between the steel beams to produce the highest flexural effects, where each of the two neoprene pads, that support the load spreader beam, was set at 3 ft from the centerline of the supporting steel beam. This arrangement provided a 6-ft spacing between the neoprene pads to simulate the LRFD HS20 truck. In order to investigate the effect of the fatigue load on the structural behavior of the joint, the following actions were taken:

1. A series of strain gages and displacement devices were installed around the joint on the top and bottom surface of the precast panels, as shown in Fig. 13. First, the full fatigue load, 42.56 kips, was applied as a static load and the strain and displacement measurements were recorded with a data acquisition system (pre-fatigue measurements).

2. The fatigue load, varying from 4.00 to 42.56 kips, was applied for 2,000,000 cycles at a rate of two cycles per second.

3. A ¾-in.-deep water pool was built around the joint covering the full width of the bridge, as shown in Fig. 12(d). The pool was continuously kept full of water before and during the fatigue load was applied. Water leakage was regularly checked at the bottom surface of the joint every 12 hours.

1uE to 8uE: Top surface strain gages
9uE to 12uE: Bottom surface strain gages
D1: Bottom surface displacment device
(same arrangement was used at the second joint)

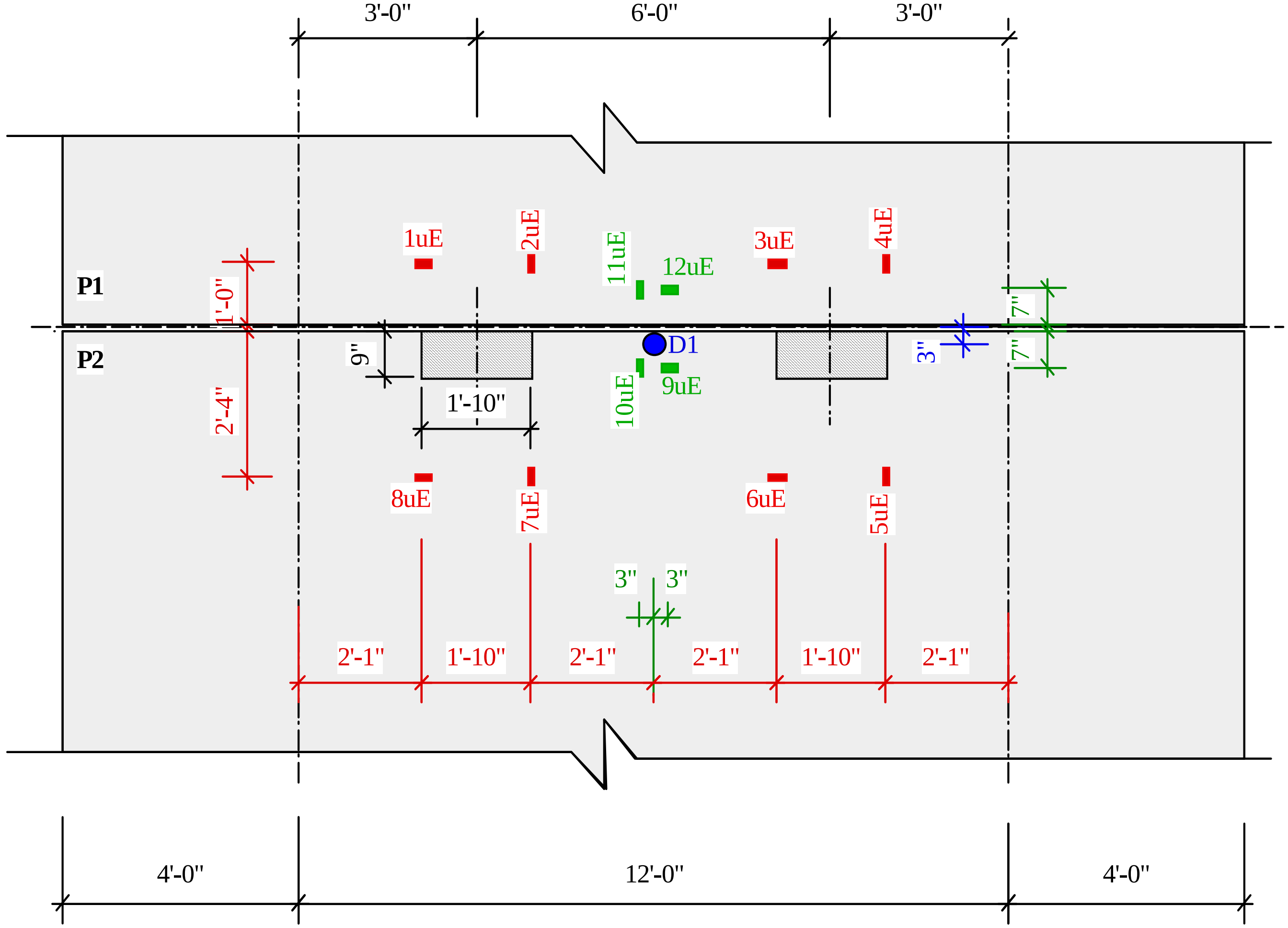


Fig. 13. Locations of the Measuring Devices
(uE = Strain Gages, D = Vertical Displacement Device)

4. Then, the full fatigue load, 42.56 kips, was applied as a static load and the strain and displacement measurements were collected (post-fatigue measurements).

5. Steps 1 to 4 were repeated at the south transverse joint P2-P3, between Panel P2 and P3

A summary of the test results is shown here below:

1. No water leakage was detected before, during or after the 2,000,000-cycle fatigue load was applied.

2. No tension cracks were observed on the bottom surface of the transverse joints or the panels after the 2,000,000-cycle fatigue load was applied. Also, no slippage occurred to the spliced No. 6 bar of CD-A and CD-B.

3. No signs of concrete crushing were observed at the top surface of the joint or the panels after the 2,000,000-cycle fatigue load was applied.

4. No separation between the grout and the vertical surface of the shear key was observed.

5. The strain measurements at P1-P2 and P2-P3 transverse joints are summarized in Figs. 14 and 15, respectively. Table 2 summarizes the displacement measurement at both joints. Studying the strain and displacement measurements revealed that:

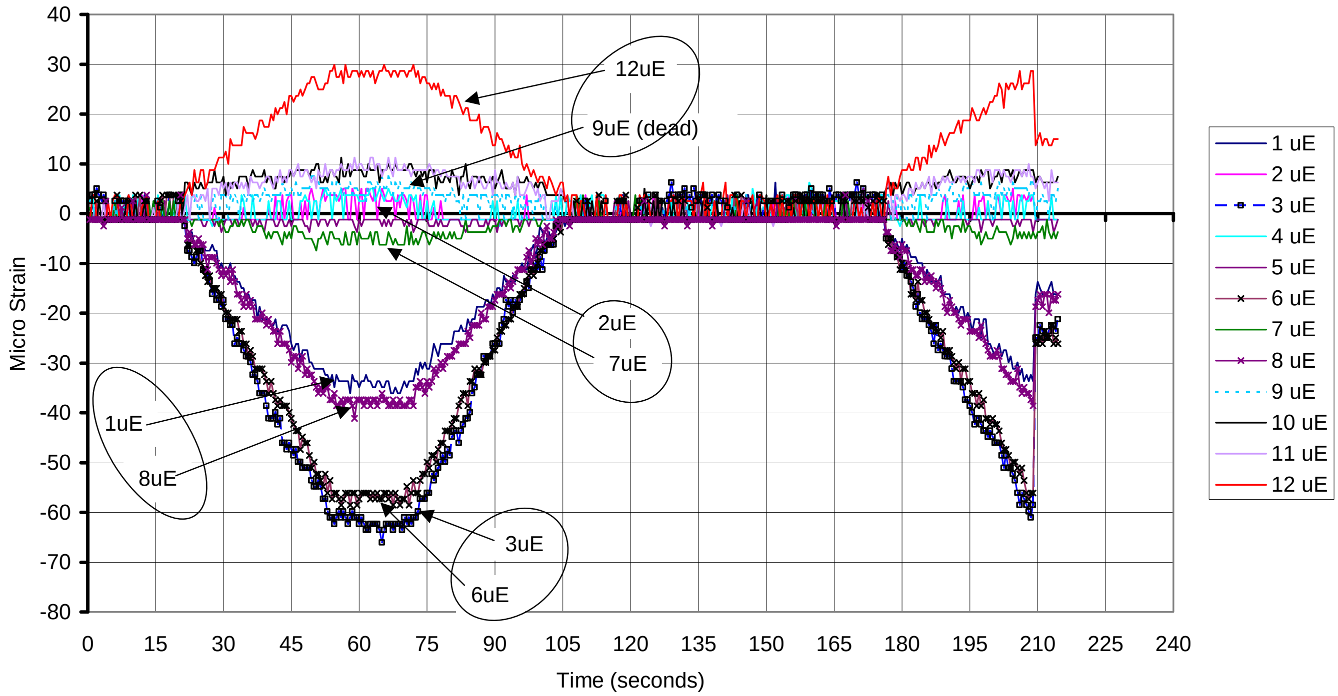
Table 2. Displacement measurements at P1-P2 and P2-P3 Joints

	Pre fatigue measurement	Post fatigue measurement
P1-P2 Joint	0.0401 in.	0.0390 in.
P1-P2 Joint	0.0379 in.	0.0388 in.

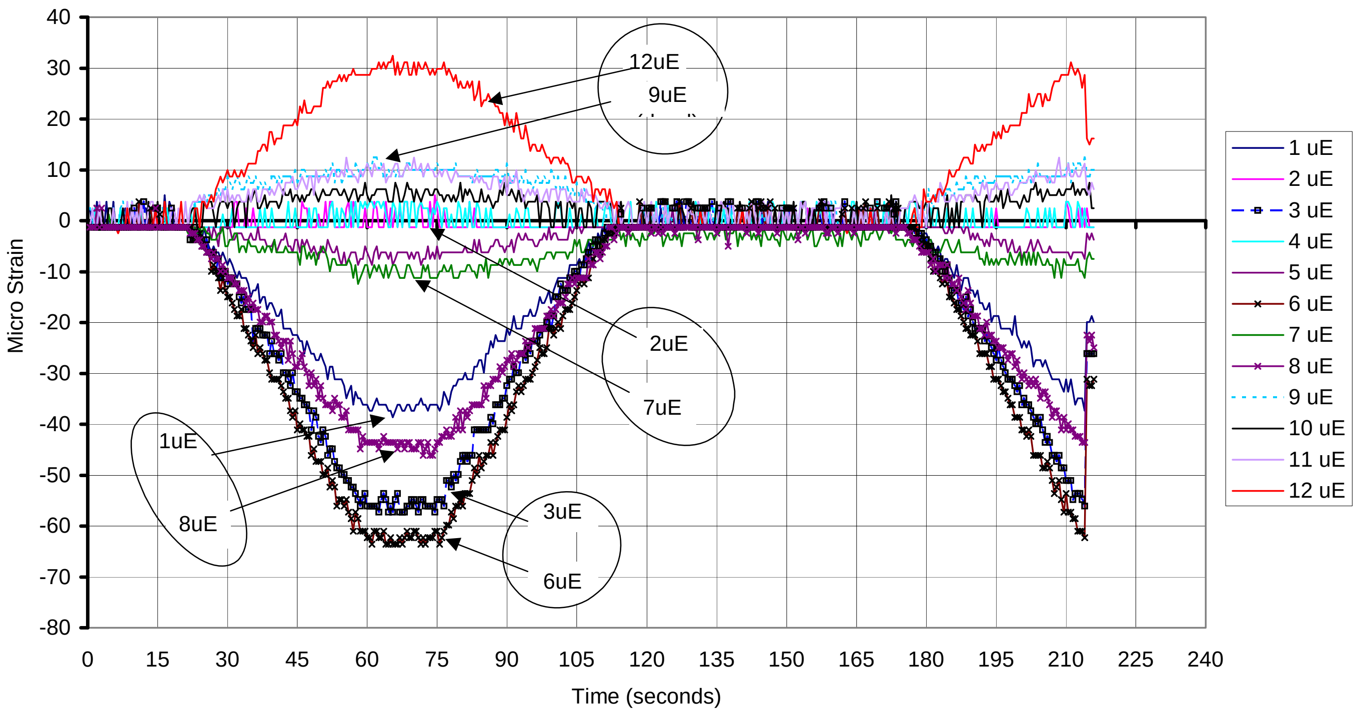
5.1 The strain gages oriented in the transverse direction (1uE, 8uE, 3uE, 6uE, 9uE &12uE) showed high stresses compared to the strain gages oriented in the longitudinal direction (2uE, 7UE, 4uE, 5uE, 10uE and 11uE). This observation confirms the logic that is used by the *Equivalent Strip Method* of the LRFD Specifications, where the deck slab is assumed to act as a one-way slab in the transverse direction.

5.2 Comparable gages on the sides of each joint showed almost the same amount of transverse strains (compare 1uE with 8uE, 3uE with 6uE, and 9uE with 12uE). This observation showed that the both joints (CD-A and CD-B) were fully able to transfer the applied load.

5.3 The strain measurements on the north and south sides of the P1-P2 and P2-P3 joints were almost identical. This observation showed that the structural behavior of the deck system was not affected by type of the panel-to-panel connection as long as the connection is capable of transferring the full load.

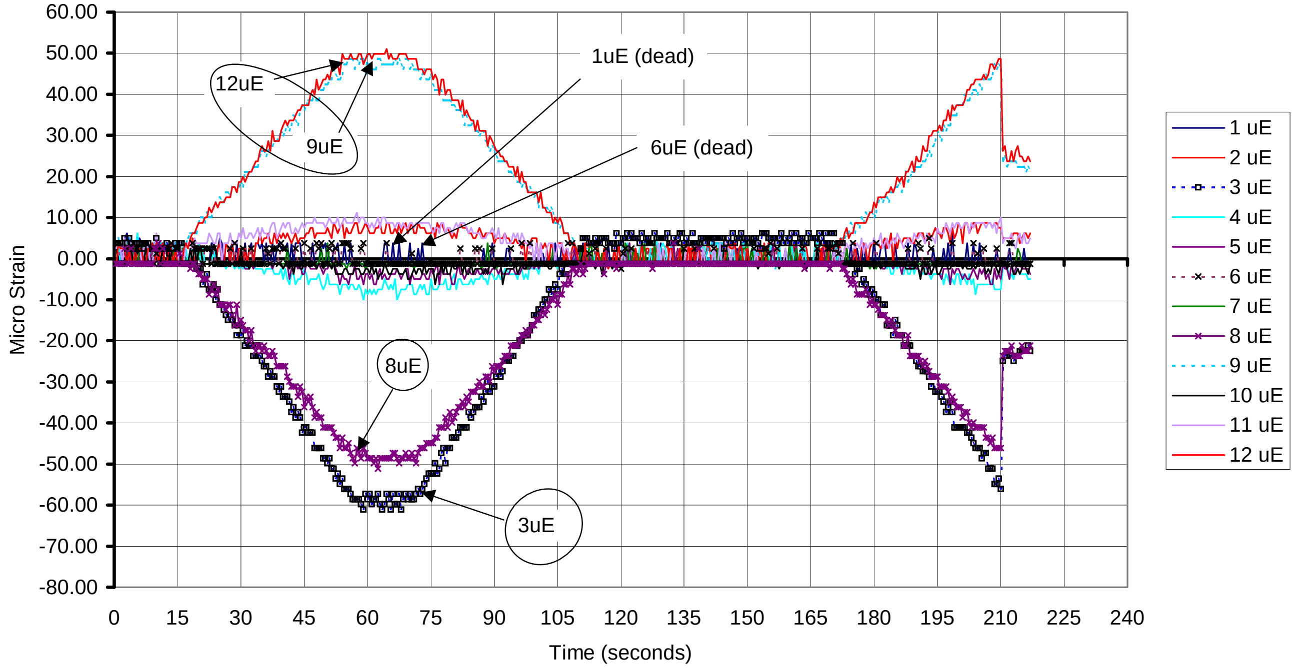


(a) Pre Fatigue Measurements

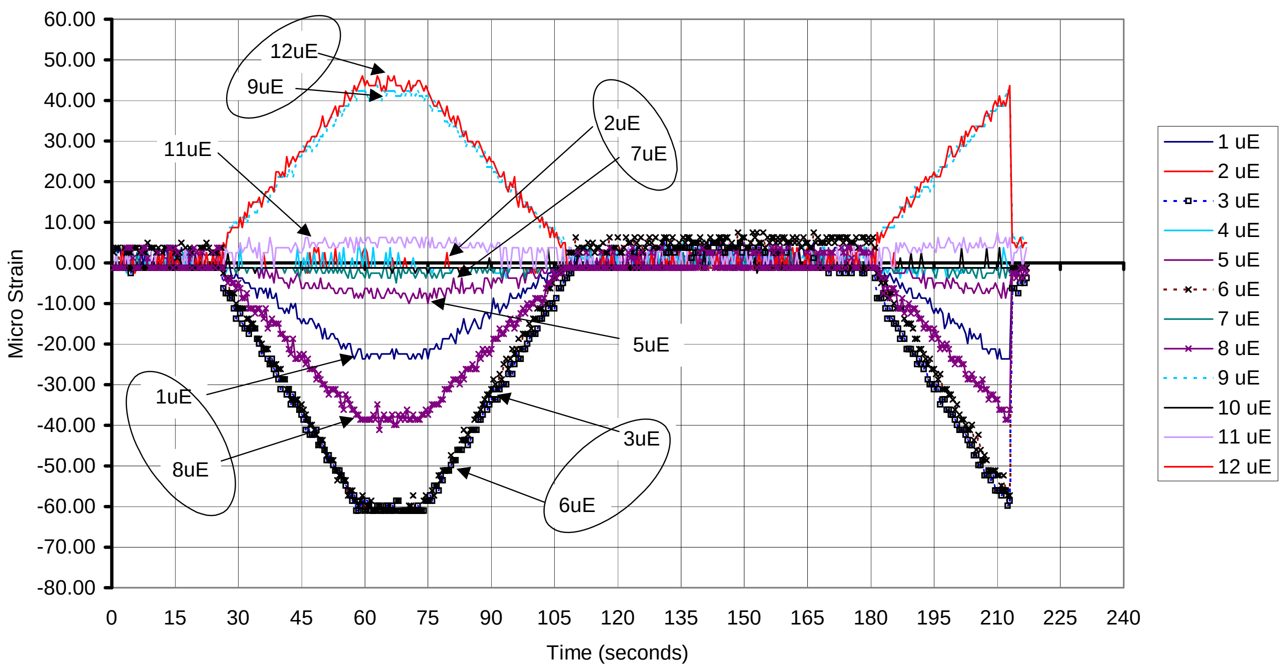


(B) Post Fatigue Measurements

Fig. 14. P1-P2 Joint, Connection Detail CD-A



(a) Pre Fatigue Measurements



(B) Post Fatigue Measurements

Fig. 15. P2-P3 Joint, Connection Detail CD-B

5.4 The stress and displacement measurements of both joints before and after the 2,000,000 cycles of fatigue load were almost the same, which indicated that no stiffness deterioration occurred due to the fatigue load.

5.5 Comparing the strain and displacement measurements of this test with those calculated using the *Equivalent Strip Method* of the LRFD Specifications showed that the LRFD equation used to calculate the width of the equivalent strip leads to a conservative design, as it distributes the wheel load on a smaller distance than what it should be, which results in higher computed flexural stresses. This observation may be due to the fact that the panels used in this test were transversely pretensioned. Transverse pretensioning increases the panel stiffness, which causes the wheel load to be distributed on a wider strip. Effect of transverse pretensioning is not recognized by the LRFD Specifications as the same equation is used to calculate the equivalent width of the strip for cast-in-place and precast concrete slabs.

CONCLUSIONS

The research developed non-prestressed panel-to-panel connection details. The connection details utilize Hollow Structural Steel (HSS) tubes to confine the grout around the spliced bars. The confinement provided by the HSS tubes significantly reduces the lap splice length. The connection details were tested for static and fatigue loads to observe and check their structural performance, and the following conclusions were reached:

1. Full-depth precast concrete panels can be effectively connected with conventional reinforcing bars.

2. Bar splice length can be significantly reduced through use of Hollow Structural Steel (HSS) tubes, which effectively confine the grout surrounding the bars. In this research, two panel-to-panel connection details were successfully developed utilizing a 4-in. (102 mm) long cut of an HSS 4 in. x12 in. x3/8 in. tube, with the following details:

    - ❖ The first connection detail requires threading a No. 6 (19) reinforcing bar, which extends about 7½ in. outside the panel to be installed, and into the old panel, which results in a 6-in. bar embedment length. The testing program has shown that this embedment distance is adequate to develop the bar yield strength. However, this connection detail requires tilting of the panel to be installed.

    - ❖ The second connection detail allows vertical installation of the new panels, where a No. 6 bar is embedded 11 in. in the HSS tube, in each of the mating joints. After a new panel is installed, a 24-in. long No. 6 splice bar is dropped through a vertical slot, which results in an 11 in. (280 mm) splice length. The testing program has shown that this splice length is adequate to develop the bar yield strength.

3. The connection details are able to deliver 125% of the specified yield strength of grade 60 steel.

4. The connection details are water tight.

5. The connection details are able of fully transfer the applied load between connected panels

REFERENCES

1. Biswas, Mrinmay, "Precast Bridge Deck Design Systems." Special Report, *PCI Journal,* V. 21, No. 2, March-April 1986, pp. 40-94.
2. Badie, S.S., Patel, P., Tadros, M.K., and Rose, J. "Utilization of Full-Depth, Precast Concrete Deck Panels, Past and Future," *PCI/NATIONAL BRIDGE CONFERENCE*, Oct. 17-20, 2004, Atlanta, Georgia, Paper 13.
3. Badie, S.S., Girgis, A.F., and Tadros, M.K., "Full-Depth, Precast-Concrete Bridge Deck Panel Systems," NCHRP 12-65, *TRB, National Academy of Science*, Final Report, December 2006.
4. Tadros, M. K. and Baishya, M. C., "Rapid Replacement of Bridge Decks." *NCHRP Report 407*, Transportation Research Board, Washington, D.C. (1998).
5. Badie, S. S.; Baishya, M. C, and Tadros, M. K. "NUDECK- An Efficient and Economical Precast Bridge Deck System." *PCI Journal,* Vol. 43, No. 5, September-October 1998, pp. 56-74.
6. Fallaha, S., Suc; C.; Lafferty, M. D.; and Tadros, M. K., "High Performance Precast Concrete NUDECK Panel System for Nebraska's Skyline Bridge." *PCI Journal,* Vol. 49, No. 5, September-October 2004, pp. 40-50.
7. *AASHTO LRFD Bridge Design Specifications*, American Association of State Highway and Transportation Officials, Washington, D.C., 3rd Edition (2004) with the 2005 & 2006 Interim Revisions.
8. Badie, S. S. "Extending Spacing of Horizontal Shear Connection Pockets from 24 to 48 inches for Steel Girders," *PCI/NATIONAL BRIDGE CONFERENCE*, October 22-26, 2006, Grapevine, TX, Paper No. 32.
9. Badie, S. S. "Efficient Full-Depth Bridge Deck Panels Connection Details," *American Concrete Institute (ACI) 2007 Spring Convention*, Hilton, Atlanta, GA, MONDAY, APRIL 23.
10. "Standard Practice for Laboratory Testing of Bridge Decks." ASTM D6275-2003, *American Standards of Testing and Materials (ASTM),* Washington, DC (2003).